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(पहला पुनरीक्षण)

Methods for Fixing the Capacities of Reservoirs

Part 4 Surcharge Storage

(First Revision)

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FOREWORD

This Indian Standard (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Reservoirs and Lakes Sectional Committee had been approved by the Water Resources Division Council.

A dam built across a river creates a reservoir behind it to store a part of the run-off from the catchment upstream of the dam. Run-off is stored when it is in excess of the demand. Demand of water for various purposes like irrigation, power, water supply etc, is met from the reservoir storage during the lean periods, when demand exceeds the run-off.

Reservoir storage generally consists of three parts as follows:

a) Inactive storage including dead storage,
b) Active or conservation storage, and
c) Flood and surcharge storage.

This standard (Part 4) was first published in 1971. This revision addresses the subject in the light of advances made from the time of the first publication. Methods for estimation of design flood have been rationalized and changes made in actual practice have been reflected. Additionally, the estimation of design flood has been treated holistically. Detailed updating of annexures has been undertaken and the latest practices detailed to make the standard more useful. Reservoir routing which was not covered earlier has been detailed in the revision along with an example of the usage of the method.

This standard consists of four parts and the other parts are as follows:

a) (Part 1) : 1999 General requirements
b) (Part 2) : 1994 Dead storage
c) (Part 3) : 2003 Live storage

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 1960 ‘Rules of rounding off numerical values (*revised*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

METHODS FOR FIXING THE CAPACITIES OF RESERVOIRS

PART 4 SURCHARGE STORAGE

(First Revision)

1 SCOPE

1.1 This standard (Part 4) covers the criteria and procedure to be followed in estimating the surcharge storage capacity of a reservoir.

2 REFERENCES

The following standards contain provisions, which through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
5542 : 1969	Guide for storm analysis
11223 : 1985	Guidelines for fixing the spillway capacity

3 TERMINOLOGY

For the purpose of this standard, the following definitions shall apply.

3.1 Normal Conservation Level (NCL) — The normal conservation level is the highest level of the reservoir at which water is intended to be held for various uses other than flood control.

3.2 Full Reservoir Level (FRL) — It is the highest level of the reservoir at which water is intended to be held for various uses including part or total of the flood storage without allowing any passage of water through the spillway.

3.3 Maximum Water Level (MWL) — It is the highest level to which the reservoir water will rise while passing the design flood with the spillway facilities in full operation.

3.4 Surcharge Storage — It is the storage between the full reservoir level and the maximum water level.

3.5 Probable Maximum Flood (PMF) — It is the flood that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region, and is computed by using the probable maximum storm which is an estimate of the physical upper limit to storm rainfall over the basin. This is obtained from transposition studies of the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions.

3.6 Standard Project Flood (SPF) — It is the flood that may be expected from the most severe combination of meteorological and hydrological factors that are considered reasonably characteristic of the region and is computed by using the standard project storm (SPS). While transposition of storms from outside the basin is permissible, very rare storms which are ‘not characteristic’ of the region concerned are excluded in arriving at the SPS rainfall for the basin.

3.7 Design Flood — It is the flood adopted for design purposes. It may be the probable maximum flood (PMF) or the standard project flood (SPF) or a flood corresponding to some desired frequency of occurrence depending upon the standard of security that should be provided against possible failure of the structure (*see* IS 11223).

4 GENERAL

4.1 A prerequisite essential to the safety of the dam is a decision in regard to the standard of security that should be provided against possible failure of the concerned structure during extraordinary floods. In case, failure of the structure is likely to result in loss of life or widespread property damage in addition to the damage of the structure itself as in the case of spillways of large reservoirs, a very high degree of security shall be provided against failure under the most severe flood conditions considered reasonably possible. But when loss of life is not involved, economic considerations would prevail to govern the selection of the design flood.

4.2 It should be assumed that the reservoir would be filled to the full reservoir level at the time of impingement of the design flood. Even though the contemplated plan of reservoir operation indicates that a portion of the storage capacity below the full reservoir level (FRL) probably would be available at the beginning of the design flood, the possibility of improper operation of regulating outlets as the result of incorrect flood predictions, mechanical difficulties, or negligent attendance may justify the assumption of a full reservoir at the time of impingement of the design flood. Moreover, future developments may require revisions in the original plan of reservoir operation or changes in the use of the reservoir that would increase the probability of a full reservoir at the time of impingement of the design flood.

4.3 Guidelines for fixing the spillway capacity are covered in IS 11223.

5 ESTIMATION OF DESIGN FLOOD

The design flood should be estimated using hydro-meteorological approach and by frequency analysis of the annual maximum flow series. Empirical formulae/ approximate methods should be avoided in estimating the design flood for medium and major structures.

5.1 Hydro-meteorological Method

In this approach the design flood is estimated by application of rainfall excess increments of the design storm to the basin response function [unit hydrograph (UG)] of the catchment. The following steps are followed:

5.1.1 *Estimation of Design Storm*

5.1.1.1 *Design storm depth*

Design storm depths, namely PMP, SPS or storm depths of various return periods and appropriate durations shall be as per IS 5542.

5.1.1.2 *Design storm duration*

Important parameter for deciding design storm duration are size and shape of the catchment, travel time/base period of unit hydrograph and direction of the storm movement with reference to the direction of river flow.

For all practical purposes, the UG base governs the duration of the design storm depth. For unit hydrograph having a base of 24 h or less, the design storm of one day is considered appropriate and sufficient. However, in case of small catchments with the base period of UG less than 12 h, the storm duration may generally be restricted to base period of UG unless the storage capacity of the reservoir is relatively large. In case, the UG base is more than 24 h but less than 48 h, 2-day design storm is considered. For basins having UG base of more than 48 h, a design storm duration of 3 day should in general be adopted. Storms of periods exceeding 72 h are not generally used in design flood estimation. However, for very large catchments, two different rainstorms simultaneously can be considered with sequential and spatial maximization techniques.

5.1.1.3 *Time adjustment of design rainfall*

The 1-day SPS/PMP rainfall depths would be less than 24 h SPS/PMP depths. Necessary corrections are to be adopted to convert 1-day rainfall into 24 h rainfall by studying the time distribution pattern of the observed storms in the area for which adequate self-recording rain gauge data is available. In cases where no such data is available, a factor of 1.15 is to be applied to 1-day rainfall to convert it to 24 h rainfall; however, the increase may be limited to 50 mm.

5.1.2 *Short Interval Distribution*

The distribution of design storm into short time intervals is done based on the observed time distribution of the severe storms in the catchment. If the same is not available, various regional flood estimation reports/PMP atlases can be referred to.

5.1.3 *Loss Rate*

Commonly, the initial loss and infiltration index values are derived on the basis of available hydrologic records, and minimum values are selected on the assumption that the basin would be saturated at the time of occurrence of design storm, resulting in very low loss rates. Unless these values are obtained from records of very severe storms occurring in wet soil conditions, their adoption for computation of extreme floods like SPF/PMF is not recommended. Where sufficient data is not available, it is recommended that loss rate of 1-2 mm/h depending upon catchment characteristics and nature of vegetation may be applied. The loss rate is property of the catchment and is characterized by indices such as ϕ-index and W-index.

5.1.4 *Critical Sequencing*

The effective rainfall intensities are computed by subtracting the ϕ-index (Loss rate) from the actual intensities. A critical sequence of the effective storm (that is the effective hyetograph) is estimated using two bells of 12 h each per day. An illustrative example is given as Annex A.

5.1.5 *Basin Response Function*

5.1.5.1 *Unit hydrograph*

In a gauged catchment, observed flood events (preferably with high and isolated peaks) and their corresponding rainfall can be used to derive basin response function (unit hydrograph) using standard hydrological methods.

If sufficient data is not available in the catchment, synthetic unit hydrograph may be developed, considering the catchment characteristics, using established methods such as *Snyder's method*, SCS dimensionless unit hydrograph method, methods prescribed in various regional flood estimation reports for different sub-zones of India etc.

5.1.5.2 *Limitations*

a) The unit hydrograph principle is limited to watersheds having catchment area less than 5 000 km^2. Hence, larger catchments should be divided into sub-catchments of area less than 5 000 km^2. Applying the unit hydrograph principles, the design flood hydrograph for each sub-catchment is derived separately. The design flood hydrographs of all the sub-catchments are routed through the stream and/or reservoir system to the outlet of the catchment, which gives the design flood hydrograph for the entire catchment.

b) Application of the unit hydrograph principle is not recommended for small catchments having area less than 25 km^2. For small catchments, detailed rainfall-runoff modeling should be carried out.

c) The average rainfall of catchment/sub-catchments should be estimated from a well distributed rain gauge network.

d) Application of unit hydrograph principle is not recommended for snow-fed part of the catchments.

5.1.6 *Convolution*

In convolution, the hydrograph of direct surface runoff (DSRO) is computed by following the principle of linearity and superposition applicable to unit hydrograph. For this, the UG ordinates at unit duration interval are multiplied by each of the rainfall excess ordinates (at unit duration interval) and these are added by lagging, one unit duration, at a time. The procedure is illustrated in solved example at Annex A.

5.1.7 *Base Flow*

The base flow value chosen should be characteristic of the season of the storm and preferably be based on observed flood hydrographs. If observed data is not available, base flow for adjacent basins converted to flow rate per unit area of catchment may be used. In case, no such data is available, a flat rate of 0.05 cumecs/km^2 can be adopted. The estimated base flow is uniformly added to the runoff ordinates.

5.2 Frequency Analysis Method

In this approach the design flood is estimated by probability or frequency analysis of the annual maximum flow series at the project site or a nearby location. The steps followed in this approach are as follows:

5.2.1 *Data Checking*

The processed data is to be analyzed to ensure that the fundamental assumptions of frequency analysis are satisfied. The data series is to be checked for randomness and presence of trends, jump and outliers.

5.2.1.1 *Randomness*

For probabilistic process, the sequence of occurrence of the variables involved is ignored and chance of their occurrence is assumed to follow a definite distribution in which the variables are considered to be purely random. Some of the tests for checking the randomness of the data are turning point test, run test, the difference sign test, rank correlation test and serial correlation test.

5.2.1.2 *Trend*

The data series has to be analyzed to check for the presence of trend and appropriately corrected if the trend is significant. Some of the tests for checking the significance of the presence of the trend are the Difference sign test, Regression test and Kendall rank correlation test.

5.2.1.3 *Jumps*

The presence of jump in a data series may be examined by visual inspection of the plot of the data. Further, the data series may be divided into two or more subsets and the characteristics of each of various subsets may be compared to check for the presence of jump. The significance of presence of jump in mean of the flows, if any, is generally checked with t-test.

5.2.1.4 *Outliers*

Outliers are data points, which depart significantly from the trend of the remaining data. The retention, modification, deletion of these outliers can significantly affect the statistical parameters computed from data, especially from small samples. All procedures for treating outliers ultimately require judgment involving mathematical and hydrological considerations.

5.2.2 *Probability Distributions*

Once a sample has been tested for randomness, presence of trend, outliers and appropriately corrected, it has to be analyzed to test as to which of the probability distributions is best fit to it. Thus the approach includes:

a) Fitting various probability distributions to the sample and estimation of the parameters of the distribution

There are many distributions, namely the Log-normal (with 2-parameter and 3-Parameter), Pearson Type III, Log-Pearson Type III and Gumbel distributions etc. The parameters of these distributions are estimated from the annual maximum flow series using method of moments and/or method of maximum likelihood and/or method of least squares and/ or probability weighted moments and/or L-moments.

b) Applying suitable tests to identify the distributions which provide best fit.

c) Estimating the floods of different return periods.

5.2.3 *Goodness of Fit Test*

The judgment whether or not a particular probability distribution adequately describes the probability of a given sample can be arrived at by this test. D-index, Chi-square and *Komogorov-Smirnov* tests are commonly used to compare the performance of the distributions.

5.2.4 *Confidence Intervals*

A confidence interval gives an estimated range of values which is likely to include an unknown population parameter, the estimated range being calculated from a given set of sample data. The confidence intervals at a particular significance level (say, *x* percent) indicates that in repeated applications of the technique, the frequency with which the confidence interval would contain the true parameter value is (100 – *x)* percent. A typical value of '*x*' is 0.05 which corresponds to (1 – 0.05) × 100 = 95 percent confidence limits. Confidence intervals are usually calculated for 95 percent limits but we can produce confidence intervals for other limits also, say 90, 99, 99.9 percent.

The width of the confidence interval gives us a rough idea about how uncertain we are about the unknown parameter. A very wide interval may indicate that more data should be collected before anything very definite can be said about the parameter and *vice versa*. Confidence intervals are more informative than the simple results of hypothesis tests since they provide a range of plausible values for the unknown parameter.

5.2.5 *Estimation of Design Flood*

The theoretical probability distributions provide the functional relationship between the probabilities of exceedance/return period and the magnitude of flood. Using the identified distribution based on goodness of fit tests, magnitudes of flood for different return period are computed.

A prerequisite to apply this method is that annual maximum series for a considerably long period (more than 25 years) should be available. A purely statistical approach for derivation of design flood for long recurrence intervals, several times larger than the length of record, has many limitations hence; this method should be used with caution. However, in certain cases (for example, catchments with limited rain gauge network), this method can be employed if adequate discharge data are available for the analysis. Where the data is short, either partial duration method or regional frequency technique is to be adopted as tentative approach and the results are verified and checked by hydrological approaches. Various computer programmes are available for frequency analysis.

6 DETERMINATION OF MAXIMUM WATER LEVEL

When a flood passes through a reservoir having one or more uncontrolled spillway(s), the outflow rate is dependent only on the water surface elevation of the reservoir. The amount of water stored in the reservoir is also dependent on the water surface elevation. Hence, the outflow and storage are interrelated through the water surface elevation. When a flood wave propagates through a reservoir, the water surface elevation changes with time resulting in change in storage and outflow with time. When the design flood hydrograph passes through the reservoir, the elevation of water surface increases for certain time and after attaining a certain stage, it starts falling. The maximum level attained by the water surface is referred as the maximum water level for the reservoir.

The process of computing reservoir stages, storage and rate of outflow corresponding to a particular inflow hydrograph is known as reservoir routing or reservoir flood routing for which a step by step computation procedure is employed. The *Modified Puls'* or Storage Indication method is widely used and is detailed in **5.1** and **5.2**. Other methods such as dynamic and steady state methods may also be used.

6.1 Data Requirement for Storage Indication Method

The following are the basic data required:

a) Reservoir stage-discharge and stage-storage relationships,

b) Inflow hydrograph,

c) Full reservoir level,

d) Initial outflow (controlled and uncontrolled), and

e) Initial reservoir level.

The stage-discharge relationship can be developed using the general weir formula, whereas, the stage-storage relationship can be developed from the topographic map. The design flood hydrograph may be treated as the inflow hydrograph. The controlled outflow refers to the estimated outflow through turbines, sluices etc, whereas the uncontrolled outflow is the discharge through the spillway. The full reservoir level may be treated as the initial reservoir level when reservoir routing is done for the computation of maximum water level for the design flood hydrograph.

6.2 Reservoir Routing

Mass balance is basic principle used for reservoir routing, according to which the rate of change of storage is equal to the difference in rate of inflow and outflow, that is

$$\frac{dS}{dt} = I - Q \qquad \ldots (1)$$

where, dS/dt is the rate of change of storage at time 't'; I and Q are the rate of inflow and out flow at time 't'. It may be noted that Q includes the rate of controlled and uncontrolled outflows at the instant 't'.

For a finite duration Δt, this is mathematically expressed as:

$$S_2 - S_1 = \left(\frac{I_2 + I_1}{2}\right)\Delta t - \left(\frac{Q_2 + Q_1}{2}\right)\Delta t \qquad \ldots (2)$$

where, I_1 and I_2 are the inflows at time 't' and '$t + \Delta t$' respectively, and Q_1 and Q_2 are the outflows at time 't' and '$t + \Delta t$' respectively.

Assuming that the inflow and stage of the reservoir (hence the outflow and storage) are known at the beginning of routing, the known terms of Equation (2) can be grouped on the LHS and the unknowns on the RHS:

$$\left(I_2 + I_1\right) + \left(\frac{2S_1}{\Delta t} - Q_1\right) = \left(\frac{2S_2}{\Delta t} + Q_2\right) \qquad \ldots (3)$$

To use Equation (3) for routing, the relationships of $\left(\frac{2S_1}{\Delta t} - Q_1\right)$ and $\left(\frac{2S_2}{\Delta t} + Q_2\right)$ with the outflow (Q) are required. To develop these relationships, first the time step (Δt) for routing is selected. For each outflow rate (Q), using the stage-discharge relationship, the reservoir water surface elevation is determined and subsequently used to estimate the storage (S) from the stage-storage relationship. Using the values of Q, S and Δt, the relationships are determined.

The step-by-step procedure to perform reservoir routing is as follows:

a) As the beginning of routing, the inflows (I_1 and I_2) are known; storage in the reservoir (S_1) and outflow (Q_1) can be computed from the initial water level in the reservoir using respective relationships. Compute the left hand side of Equation (3) using the known quantities and this will be equal to $\left(\frac{2S_2}{\Delta t} + Q_2\right)$.

b) Determine the outflow rate (Q_2) from the $\left(\frac{2S}{\Delta t} + Q\right)$ *versus* Q relationship.

c) Determine $\left(\frac{2S_2}{\Delta t} - Q_2\right)$ by subtracting twice the value of Q_2 determined in step (b) from $\left(\frac{2S_2}{\Delta t} + Q_2\right)$ determined in step (a) which gives the value of $\left(\frac{2S_1}{\Delta t} - Q_1\right)$ for the next time step.

d) Add I_1 and I_2 to $\left(\frac{2S_1}{\Delta t} - Q_1\right)$, determined in the previous step, which yields in $\left(\frac{2S_2}{\Delta t} + Q_2\right)$ for the current time step, using which determine the outflow at the current time step (Q_2) from the derived relationship.

e) Using the known quantities calculate the left hand side of Equation (3) for the next time step.

f) Repeat steps (d) and (e) till there is no significant change in two consecutive outflows.

g) The maximum stage attained during the entire routing period gives the maximum water level for the reservoir.

It may be observed that all the calculations are dependent on the time step (Δt). If it is too long, then the peak of the inflow hydrograph may be missed within a time step and will not be accounted for, resulting in inappropriate routing; whereas, too short time step results in a lengthy routing. Further, Δt should be small enough such that the inflow hydrograph can be treated as varying linearly during that period. It is recommended that Δt should be one third to half of the time of travel through the reservoir. As a thumb rule, Ät can be taken as 20 percent to 40 percent of the time to peak of the inflow hydrograph. As the computers are almost universally used for reservoir routing these days, Δt can be conveniently taken to be of the order of a few hours.

When $\left(\frac{Q_2}{2} > \frac{S_2}{\Delta t}\right)$, the outflow becomes negative which results in distortion in the hydrograph. To avoid such situation the selection of Ä*t* can be made as follows:

a) Select a time step using the thumb rule.

b) Develop the $\left(\frac{2S_2}{\Delta t} + Q_2\right)$ *versus* Q relationship using stage-storage and stage-discharge relationships.

c) Plot the values on a log-log paper with Q in abscissa and $\left(\frac{2S_2}{\Delta t} + Q_2\right)$ in the ordinate.

d) Compare the curve with the line of equal values. If the plotted values lie above the line of equal values, discard the selected Δt and choose a new value and repeat the procedure.

6.3 The reservoir routing procedure is elaborated in Annex B with a numerical example.

ANNEX A

(*Clauses* 5.1.4 *and* 5.1.6)

ILLUSTRATIVE EXAMPLE FOR THE PROCESS OF CONVOLUTION

The design storm studies of a project near Orissa coast gave the 1-day, 2-day and 3-day Probable Maximum Precipitation (PMP) depths as 641.4, 863.6 and 1 066.8 mm respectively. The 1-hour, 1-mm Unit Hydrograph (UG) obtained for the project is shown below. The loss rate ϕ index and base flow selected for the project site are 1.5 mm/h and 30 cumecs respectively. The catchment area up to the site is 663 sq. km.

1-hour UG Ordinates at 1-hour Interval in Cumecs:

1 hour UG ordinates at 1-hour interval in cumec

0.00	3.80	8.30	16.50	21.30	25.00	23.50	19.80	16.30	12.80	10.50
8.25	6.25	4.50	3.25	2.00	1.25	0.75	0.05	0.00	—	—

Short Interval Time Distribution for Design Storm:

Within the 12 h, short interval distribution of rainfall is obtained from Self Recording Rain Gauge (SRRG) data pertaining to storm of the same type as the PMP storm under consideration, and is given as below:

Time (h)	1	2	3	6	12
Percentage of 12 h Rainfall	22.0	36.1	45.0	67.0	100.0

Based on the SRRG data of 10 major spells of 24 h duration storms in the region, it is found that the ratio of first-12 hour depth to 24 h depth is 75 percent. With this information derive the design flood hydrograph for the 3-day storm.

Solution:

Step 1 — Critical arrangement of rainfall ordinates

As the 1-day, 2-day and 3-day PMP depths are 641.4, 863.6 and 1 066.8 mm respectively, the rainfall depths on the individual days are taken to be 641.4, 222.2and 203.2mm. These are arranged in such a way that the largest depth is preceded by the 2nd largest and succeeded by the 3rd largest. Thus the rainfall depths on the 1st, 2nd and 3rd day are 222.2, 641.4 and 203.2 mm respectively.

Step 2 — The rainfall depths are proposed to be arranged in 2 bells per day. The storm depths for 12 h blocks are determined using the 12 h to 24 h ratio of 75 percent given above, as follows:

1st day depth – 222.2 mm		*2nd day depth – 641.4 mm*		*3rd day depth – 203.2 mm*	
1st 12 h (0.75)	2nd 12 h (0.25)	1st 12 h (0.75)	1st 12 h (0.25)	1st 12 h (0.75)	2nd 12 h (0.25)
166.65	55.55	481.05	160.35	152.40	50.80

Step 3 — Using the distribution percentages given above, cumulative depths and incremental depths for unit duration was obtained and are given in Table 1 and Table 2.

Table 1 Cumulative Rainfall at Unit Duration

Time h	**Percentage**	**Cumulative Rainfall in each Bell,** mm					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	22.0	36.7	12.2	105.8	35.3	33.5	11.2
2	36.1	60.2	20.1	173.6	57.9	55.0	18.3
3	45.0	75.0	25.0	216.5	72.1	68.6	22.9
4	53.7	89.5	29.9	258.3	86.1	81.8	27.3
5	60.9	101.5	33.9	292.9	97.6	92.8	30.9
6	67.0	111.7	37.3	322.3	107.4	102.1	34.0
7	72.9	121.5	40.5	350.6	116.9	111.1	37.0
8	78.6	131.0	43.7	378.1	126.0	119.8	39.9
9	84.1	140.2	46.8	404.5	134.8	128.2	42.7
10	89.5	149.2	49.8	430.5	143.5	136.4	45.5
11	94.8	158.0	52.7	456.0	152.0	144.5	48.2
12	100.0	166.7	55.6	481.0	160.3	152.4	50.8

Table 2 Rainfall Increments at Unit Duration

Time h	**Percentage**	**Rainfall Increments in each Bell,** mm					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	22.0	36.7	12.2	105.8	35.3	33.5	11.2
2	14.1	23.5	7.8	67.8	22.6	21.5	7.2
3	8.9	14.8	4.9	42.8	14.3	13.6	4.5
4	8.7	14.5	4.8	41.8	13.9	13.3	4.4
5	7.2	12.0	4.0	34.6	11.5	11.0	3.7
6	6.1	10.2	3.4	29.3	9.8	9.3	3.1
7	5.9	9.8	3.3	28.4	9.5	9.0	3.0
8	5.7	9.5	3.2	27.4	9.1	8.7	2.9
9	5.5	9.2	3.1	26.5	8.8	8.4	2.8
10	5.4	9.0	3.0	26.0	8.7	8.2	2.7
11	5.3	8.8	2.9	25.5	8.5	8.1	2.7
12	5.2	8.7	2.9	25.0	8.3	7.9	2.6

Step 4 — The above incremental depths are arranged in critical order for each bell separately. For this the largest of increments is placed against the peak of UG, then the next largest against the next largest of UG ordinate and so on until all rainfall increments get arranged and then the sequence is reversed to get the critical sequence. The same procedure is repeated for other spells also. The resulting series are given in Table 3 and Table 4 respectively.

Table 3 Critical Sequence of Rainfall at Unit Duration

Time h	UG Ordinates	**Critical Sequence of Rainfall Increments in each Bell,** mm					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.0	3.80	9.2	3.1	26.5	8.8	8.4	2.8
2.0	8.30	12.0	4.0	34.6	11.5	11.0	3.7
3.0	16.50	14.8	4.9	42.8	14.3	13.6	4.5
4.0	21.60	36.7	12.2	105.8	35.3	33.5	11.2
5.0	25.00	23.5	7.8	67.8	22.6	21.5	7.2
6.0	23.50	14.5	4.8	41.8	13.9	13.3	4.4
7.0	19.80	10.2	3.4	29.3	9.8	9.3	3.1
8.0	16.30	9.8	3.3	28.4	9.5	9.0	3.0
9.0	12.80	9.5	3.2	27.4	9.1	8.7	2.9
10.0	10.50	9.0	3.0	26.0	8.7	8.2	2.7
11.0	8.25	8.8	2.9	25.5	8.5	8.1	2.7
12.0	6.25	8.7	2.9	25.0	8.3	7.9	2.6
4.50							
3.25							
2.00							
1.25							
0.75							
0.05							
0.00							

Table 4 Reverse Critical Sequence

Time h	**Critical Sequence of Rainfall Increments after Interchange of 1st and 2nd Spells,** mm					
	1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.0	8.7	2.9	25.0	8.3	7.9	2.6
2.0	8.8	2.9	25.5	8.5	8.1	2.7
3.0	9.0	3.0	26.0	8.7	8.2	2.7
4.0	9.5	3.2	27.4	9.1	8.7	2.9
5.0	9.8	3.3	28.4	9.5	9.0	3.0
6.0	10.2	3.4	29.3	9.8	9.3	3.1
7.0	14.5	4.8	41.8	13.9	13.3	4.4
8.0	23.5	7.8	67.8	22.6	21.5	7.2
9.0	36.7	12.2	105.8	35.3	33.5	11.2
10.0	14.8	4.9	42.8	14.3	13.6	4.5
11.0	12.0	4.0	34.6	11.5	11.0	3.7
12.0	9.2	3.1	26.5	8.8	8.4	2.8

Step 5 — Now, the 1st 12 h block is interchanged with the next 12 h block to create the critical situation as shown in Table 5.

Table 5 Interchange of 1st and 2nd 12 h Blocks (Bell)

Time h	**Critical Sequence of Rainfall Increments after Interchange of 1st and 2nd Spells,** mm					
	1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.0	2.9	8.7	25.0	8.3	7.3	2.6
2.0	2.9	8.8	25.5	8.5	8.1	2.7
3.0	3.0	9.0	26.0	8.7	8.2	2.7
4.0	3.2	9.5	27.4	9.1	8.7	2.9
5.0	3.3	9.8	28.4	9.5	9.0	3.0
6.0	3.4	10.2	29.3	9.8	9.3	3.1
7.0	4.8	14.5	41.8	13.9	13.3	4.4
8.0	7.8	23.5	67.8	22.6	21.5	7.2
9.0	12.2	36.7	105.8	35.3	33.5	11.2
10.0	4.9	14.8	42.8	14.3	13.6	4.5
11.0	4.0	12.0	34.6	11.5	11.0	3.7
12.0	3.1	9.2	26.5	8.8	8.4	2.8
Total	55.6	166.7	481.0	160.3	152.4	50.8

Maximum consecutive 24 h value = 647.7

Step 6 — The maximum depth for any duration within this sequence shall not exceed the design storm depth for that duration. Specifically, the maximum depth in any 24 h sequence shall be checked to ensure that this is not more than the design storm depth for 24 h.

In this case, the maximum 24 h depth in the design hyetograph is found to be equal to 647.7 mm. The 24 h storm depth (by applying clock hour correction to 1-day depth) is equal to 1.15 × 641.4 = 724 mm or 641.4 + 50 mm whichever is less that is 691.4 mm. Therefore no modification is essential.

Step 7 — Now, the loss rate (f-index) of 1.5 mm/h is uniformly deducted from the rainfall increment values given in Table 5 obtain rainfall excess ordinates as shown in Table 6 and Fig. 1.

Table 6 Adopted Sequence of Rain Fall Excess at Unit Duration

Time h	**Critical Sequence of Rainfall Increments in each Bell,** mm					
	1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.0	1.4	7.2	23.5	6.8	6.4	1.1
2.0	1.4	7.3	24.0	7.0	6.6	1.2
3.0	1.5	7.5	24.5	7.2	6.7	1.2
4.0	1.7	8.0	25.9	7.6	7.2	1.4
5.0	1.8	8.3	26.9	8.0	7.5	1.5
6.0	1.9	8.7	27.8	8.3	7.8	1.6
7.0	3.3	13.0	40.3	12.4	11.8	2.9
8.0	6.3	32.0	66.3	21.1	20.0	5.7
9.0	10.7	35.2	104.3	33.8	32.0	9.7
10.0	3.4	13.3	41.3	12.8	12.1	3.0
11.0	2.5	10.5	33.1	10.0	9.5	2.2
12.0	1.6	7.7	25.0	7.3	6.9	1.3

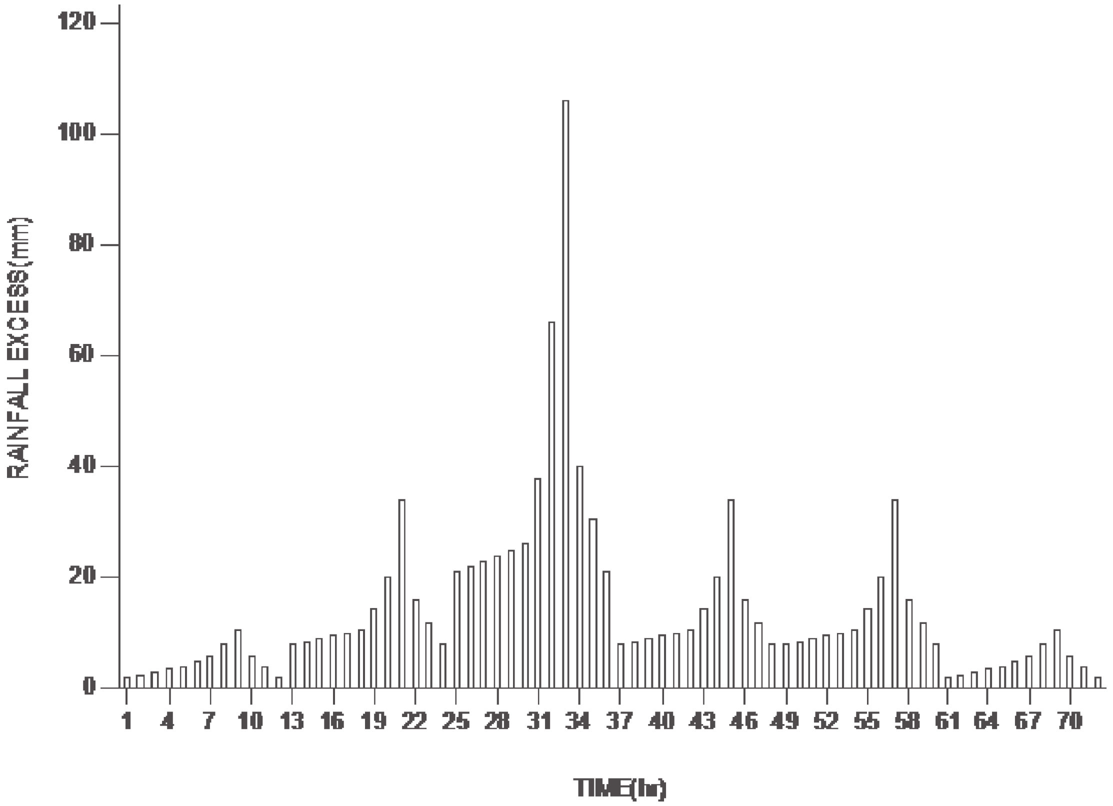

Fig. 1 Excess Hyetograph (Adopted)

Step 8 — Convolution

a) Multiply each excess rainfall ordinate with the unit hydrograph ordinates.

b) Add the resulting individual DSRO's by lagging one unit duration at a time, to the total DSRO. Many computer programmes are available to conveniently do this.

c) Add base flow uniformly to the DSRO ordinates to get the design hydrograph coordinates. The calculations are shown in Table 7. The resultant design hydrograph is shown in Fig. 2.

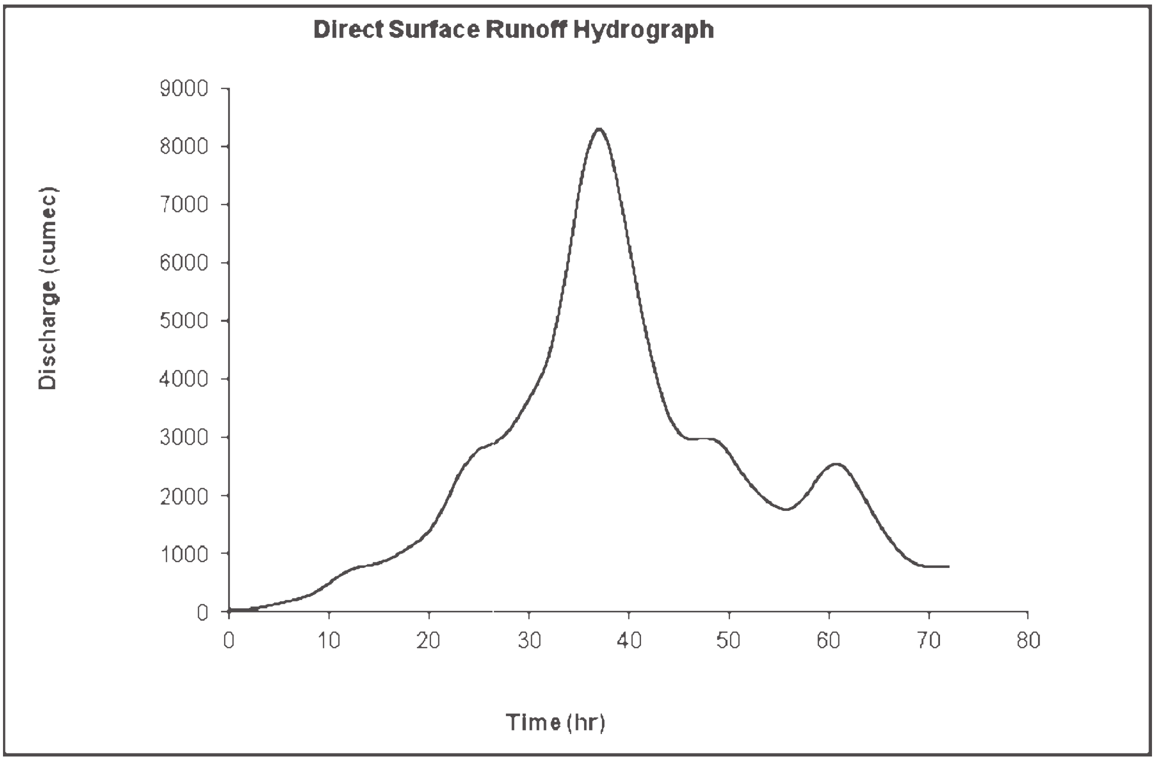

FIG. 2 DESIGN FLOOD HYDROGRAPH

The following checks may be carried out before finally adopting the design hydrograph.

a) Check for volume of DSRO — The volume of DSRO (converted to depth over catchment area) shall be equal to the depth of rainfall excess hyetograph. If not, there is some calculation error, which should be rectified. In this case, depth of rainfall excess = 958.8 mm and volume of DSRO

$$\frac{176\,600 \times 1 \times 3.6}{663} = \mathbf{958.8\ mm}$$. So it's OK.

b) Check the ratio of total volume of DSRO to total rainfall depth for its reasonableness considering the shape, size and location of the catchment.

c) Check the peak of the design hydrograph for regional consistency by comparing with values adopted for catchments of comparable size and shape in the region.

ANNEX B

(*Clause* 5.3)

NUMERICAL EXAMPLE OF RESERVOIR ROUTING

The elevation-capacity-spillway release capacity for a reservoir is given in Table 8. The ordinates of a major flood hydrograph are given in Table 9 at 2 h interval. Route the flood using the *Modified Puls'* method assuming the initial reservoir elevation at 180.0 m.

Solution: The input data are given in col 2, 3 and 4 of Table 8. Using these, col 5 and 6 can be easily computed.

Table 8 Elevation-Capacity-Spillway Release Capacity for the Dam

Sl No. (1)	**Elevation** (m) (2)	**Storage** S (10^6 m^3) (3)	**Spillway Release Capacity** Q (m^3/s) (4)	$[(2S/\Delta t)+Q]$ (m^3/s) (5)	$[(2S/\Delta t)-Q]$ (m^3/s) (6)
1	170.69	29.078	0.0	8 077.22	8 077.22
2	173.74	58.898	0.0	16 360.56	16 360.56
3	176.78	103.203	0.0	28 667.50	28 667.50
4	178.92	157.178	0.0	43 660.56	43 660.56
5	179.83	180.844	279.23	50 513.67	49 955.21
6	182.88	304.596	2 704.73	87 314.73	81 905.27
7	185.93	497.225	6 718.83	144 836.89	131 399.23
8	188.98	763.135	12 022.37	224 004.31	199 959.57
9	189.59	829.415	13 225.70	243 618.76	217 167.36
10	190.50	926.847	15 095.94	272 553.44	242 361.56
11	192.02	1 108.144	18 427.08	326 244.86	289 390.70
12	193.55	1 309.163	21 982.90	385 639.29	341 673.49
13	194.00	1 420.000	23 400.00	417 844.44	371 044.44

The results of computation for the initial reservoir elevation of 180.0 m by following the steps given earlier are shown in Table 9. It can be seen from this table that the peak of the outflow hydrograph was 18 809 m^3/s while the peak of the inflow hydrograph was 27 180 m^3/s. The peak of inflow occurred at 49 h while for outflow, it was at 57 h. The maximum water level attained by the reservoir was 192.18 m.

Table 9 Flood Routing Through a Reservoir Using the Modified Puls' Method

Sl No. (1)	**Time** *h* (2)	**Reservoir Elevation,** *m* (3)	**Inflow** m^3/s (4)	**Reservoir Storage** (10^6 m^3) (5)	**Outflow** m^3/s (6)
i)	1	180.00	566.41	187.74	414.42
ii)	3	180.05	854.72	189.73	453.46
iii)	5	180.18	1 598.98	194.93	555.40
iv)	7	180.43	2 514.02	205.03	753.27
v)	9	180.79	3 438.12	219.98	1 046.28
vi)	11	181.27	4 337.86	239.09	1 420.86
vii)	13	181.82	5 189.46	261.57	1 861.50
viii)	15	182.44	6 000.57	286.68	2 353.64
ix)	17	183.02	6 746.53	313.68	2 894.04
x)	19	183.47	7 414.61	341.72	3 478.34
xi)	21	183.92	7 981.87	369.98	4 067.30
xii)	23	184.36	8 452.56	397.78	4 646.49
xiii)	25	184.78	8 848.77	424.60	5 205.34
xiv)	27	185.18	9 162.28	450.05	5 735.73
xv)	29	185.57	9 576.04	474.38	6 242.83
xvi)	31	185.95	10 231.96	498.91	6 752.41
xvii)	33	186.25	11 034.55	524.98	7 272.38
xviii)	35	186.58	12 020.11	553.56	7 842.47
xix)	37	186.94	13 231.95	585.70	8 483.38
xx)	39	187.37	14 825.54	622.95	9 226.37
xxi)	41	187.89	17 065.99	668.09	10 126.65
xxii)	43	188.53	19 758.99	723.75	11 236.81
xxiii)	45	189.25	23 102.24	792.40	12 553.74
xxiv)	47	190.01	26 187.76	873.96	14 080.81
xxv)	49	190.77	27 180.12	958.93	15 685.41

Table 9 — (*Concluded*)

Sl.No. (1)	**Time** *h* (2)	**Reservoir Elevation**, *m* (3)	**Inflow** m^3/s (4)	**Reservoir Storage** (10^6 m^3) (5)	**Outflow** m^3/s (6)
xxvi)	51	191.39	26 113.85	1 032.96	17 045.57
xxvii)	53	191.84	23 965.73	1 086.94	18 037.53
xxviii)	55	192.10	21 500.71	1 118.68	18 613.42
xxix)	57	192.18	18 990.37	1 129.73	18 808.84
xxx)	59	192.13	16 425.38	1 122.27	18 677.02
xxxi)	61	191.94	13 922.40	1 098.58	18 251.41
xxxii)	63	191.63	11 504.96	1 061.19	17 564.27
xxxiii)	65	191.22	9 310.96	1 012.86	16 676.28
xxxiv)	67	190.75	7 436.99	956.79	15 646.09
xxxv)	69	190.21	5 883.32	896.19	14 507.45
xxxvi)	71	189.63	4 683.09	834.07	13 315.01
xxxvii)	73	189.07	3 701.22	772.43	12 191.10

FIXING TOP OF DAM

The top of a dam is fixed a little above the high flood level by providing extra space known as freeboard. Freeboard is the vertical distance between the maximum water level and the top of the dam. Freeboard is provided to ensure the safety of the dam against overtopping in the event of an adverse combination of hydro-meteorological variables, such as strong winds during a major storm which may push up the reservoir water level. The size of the freeboard depends on the design flood for freeboard, wind set-up, and the wave run-up that is expected depending on the length of the reservoir, prevailing winds, etc. When wind blows over the surface of a reservoir, water is piled up at the leeward end and is lowered at the windward end. The set-up is more pronounced in shallow reservoirs and can be calculated by:

$$H_s = kn\frac{u^2}{gd} l \times \cos\theta$$

where

H_s = is the height of set-up above still pool level,

u = the wind speed measured at an elevation of 10m above the still pool level,

l = the fetch length (straight length of unobstructed water surface exposed to wind action), n is a dimensionless coefficient dependent upon the configuration and hydrology of lake,

q = the angle between wind direction and line along which fetch is measured,

g = acceleration due to gravity,

d = average water depth along the direction of wind, and

k = a dimensionless shear-stress coefficient.

For rectangular lakes of uniform depth, $n = 1$ and $k = 1.45 \times 10^{-6}$ provided $u \leq 880\ d\left(\frac{g}{l\cos\theta}\right)$.

When wind blows across reservoirs, waves are generated which run up on the upstream face of the dam. This run up is critical only when reservoir water level is near the top. Adequate freeboard is required to prevent overtopping of a dam by waves.

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